CHAPTER IX
HYDRAULICS AND DRAINAGE

SECTION 9-02 HYDROLOGIC ANALYSIS

9-02.1 GENERAL. Hydrology is that branch of the applied earth sciences which deals with the occurrence and distribution of surface and ground waters. Highway drainage design is concerned with the surface water hydrology of small watersheds. In particular, the peak rate of runoff and/or the runoff hydrograph produced by flood events are of interest in analysis of highway drainage problems. Peak runoff rates are a parameter of design and their estimation is a prerequisite to hydraulic computations. Flood hydrographs are a necessary parameter of stormwater detention basin design, which can be used to reduce peak runoff rates for design of certain highway drainage structures.

In general, the design of drainage facilities may be described as follows:

- Select the design frequency
- Estimate the peak rate of runoff or runoff hydrograph resulting from the design and review floods
- Design the drainage structure to pass the design flood peak or runoff hydrograph
- Check the hydraulic capacity of the proposed system for the base flood
- Check that the final design meets the requirements of the National Flood Insurance Program (NFIP)

Criteria for determination of the design flood frequency and methods for computing peak rates of runoff and runoff hydrographs are presented in this section.

9-02.2 DESIGN FREQUENCY AND RETURN PERIOD. The frequency of occurrence or return period of an event may be defined as the average period of time between events equal to or greater than a given magnitude. The annual probability of occurrence of an event is equal to the reciprocal of the event's return period. For example, a flood with a return period of 100 years has a 1% chance of occurring in any year; whereas a flood with a return period of 25 years has a 4% chance of occurring in any year.

9-02.2 (1) DESIGN FREQUENCY CRITERIA. The return period used as a criterion of design is known as the design frequency. The design frequency to be used in the design of drainage structures is a function of traffic volume and type of drainage facility, as given in Table 9-02.1. The design flood magnitude, design frequency and corresponding water surface elevation(s) shall be included on the project plans for all crossroad structures.

The design frequency for roadside drainage ditches should be chosen based on the function served. Ditches with a primary function of removal of water from the pavement should be based on the design frequency of the pavement drainage facilities. Ditches with a primary function of carrying water to or from a crossroad structure should be based on the design frequency of the crossroad structure.

TABLE 9-02.1
DESIGN FREQUENCY

ADT	CROSSROAD DRAINAGE	PAVEMENT DRAINAGE
0-400	10-year	5-year
400-1700	10 to 25 year	5-year
1700-5000	25-year	10-year
Over 5000	50-year	10-year
Interstate	50-year	10-year

9-02.2 (2) BASE FLOOD AND OVERTOPPING FLOOD. The base flood is defined as the 100-year flood or the flood with a 1% chance of being exceeded in any given year. The overtopping flood is defined as the discharge and corresponding return period and water surface elevation at which flow occurs over the roadway.

After sizing a drainage facility using the design flood corresponding to the return period from Table 9-02.1, the ability of the proposed facility to pass the base (100-yr) flood shall also be evaluated, with special attention to any risks to people or property. This is done to ensure that there are no unexpected flood hazards inherent in the proposed facility for flood events exceeding the design discharge.

The overtopping flood shall be determined for all crossroad structures, and the resulting overtopping flood magnitude and approximate return period shall be included on the project plans. When the overtopping flood is greater than the 100-year flood, the return period should be noted as ">100 yr". If the overtopping flood is less than the 100-year flood, the magnitude and water surface elevation of the 100-year flood shall also be included on the project plans.

9-02.2 (3) NATIONAL FLOOD INSURANCE PROGRAM CRITERIA. Meeting the requirements of the National Flood Insurance Program (NFIP) regulations can necessitate larger structures than those designed to meet the above design frequencies. The designer shall ensure that the final design meets all requirements of the NFIP. Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS) maps should be consulted to determine the applicable NFIP requirements. Development in a mapped regulatory floodway can cause no increase in Base Flood Elevations (BFE's). Development in a mapped Special Flood Hazard Area where no floodways have been defined can cause no more than one-foot [0.30 m] increase in BFE's. See Section 4-09.21 (4) for definitions and additional information on the NFIP.

9-02.3 DESIGN DATA. In order to carry out the hydrologic analysis of a watershed, it is necessary to assemble certain data. This data includes the drainage area, the length of the hydraulically longest drainage path, the elevation of the watershed ridge, the elevation of the watershed outlet, the hydrologic soil group, the type of terrain, the land use of the watershed, and information on the extent of development in urban areas. The topographic data should be obtained from suitable maps when maps are available. Suitable maps are defined as department manuscripts and 7-1/2 or 15 minute USGS topographic maps or maps of equal quality. Only when such maps are not available should field measurements be made for the purpose of obtaining hydrologic information. The land use and terrain of the watershed should be evaluated in the field by the designer. The hydrologic soil group is either given on the soil survey performed by the Materials Division or is to be determined from county soil survey maps or from information from the soil survey and site visits.

9-02.4 RURAL VS. URBAN HYDROLOGY. Small watersheds may be divided into rural and urban classifications. A rural watershed is a drainage basin whose natural response to rainfall has not been substantially altered by urban land activity. Rural watersheds may be either natural or agricultural.

An urban watershed may be defined as a drainage basin in which manmade developments in the form of impervious surfaces and/or storm drainage systems have substantially altered the basin's natural response to rainfall. Urbanization of a natural watershed progresses in one of two ways. First, the addition of impervious surfaces in the form of roads, streets, parking lots and roofs will prevent infiltration of rainfall into the covered soil surface, thus increasing the total volume and peak rate of runoff from a given rainfall volume. Second, to protect the now valuable property in a developed watershed from this increased peak and volume of runoff, storm drainage systems are installed. The installation of a storm drainage system does not increase the volume of runoff, but modifies the time distribution of runoff. Thus, when storm water drainage systems are installed, the time of concentration of the watershed is decreased. Therefore, storm water drainage systems have the effect of removing a given volume of runoff in a shorter period of time, thus further increasing the peak rate of runoff.

All hydraulic design in urban areas should consider the effect of increasing development throughout the projected life of the structure. Information on planned future development may be available from local agencies.

Two methods are presented in this section for estimating peak rates of runoff from small watersheds. The Rational Method should be used on all watersheds less than 200 acres [0.8 km^2] in size. On watersheds greater than 200 acres [0.8 km^2] in size, the USGS Regression Equations should be used. Watersheds that lie within Region III for the USGS Rural Regression Equations and have drainage areas between 200 to 300 acres [0.8 to 1.24 km^2] do not fall within the limits for either set of equations. In this case, both sets of equations should be calculated, and the designer should decide which is appropriate.

9-02.5 THE RATIONAL METHOD. The Rational Method was developed as early as 1889, and despite its limitations is one of the most widely used methods of estimating peak flows. Several assumptions are implicit in application of the Rational Method:

- The maximum runoff rate occurs when the rainfall intensity lasts as long or longer than the time of concentration
- The frequency of the discharge is the same as that of the rainfall intensity
- The fraction of the rainfall that becomes runoff is independent of the rainfall intensity or volume

The first assumption implies that a homogeneous rainfall event is applied uniformly to the entire drainage area, and may not be valid for larger watersheds where constant rainfalls of high intensity do not occur simultaneously over the entire watershed. This assumption also provides the basis for using the watershed's time of concentration as the duration of the design storm. The second assumption again limits the size of the drainage area because for larger basins, factors other than rainfall frequency can play a large role in determining the flood frequency. Finally, the third assumption is reasonable for highly impervious areas, but less reasonable for pervious areas where the antecedent moisture condition plays a large role in determining the amount of rainfall that becomes surface runoff. For these reasons, use of the Rational Method is limited to small watersheds.

9-02.5 (1) EQUATION. The Rational Method is expressed by the following formula.

$$Q = k_c \cdot C \cdot I \cdot A \tag{1}$$

where:

Q = the peak rate of runoff, ft^3/s [m^3/s]
k_c= 1.0 - ENGLISH [0.00278 - METRIC]
C = runoff coefficient, representing the ratio of direct runoff to rainfall
I = rainfall intensity of the design storm, in/hr [mm/hr]
A = drainage area of the watershed, acres [hectares]

9-02.5 (2) RUNOFF COEFFICIENT. The runoff coefficient represents the ratio of runoff to the total rainfall and combines the effects of several watershed characteristics such as land use, soil type, cover condition, and general terrain or watershed slope.

9-02.5 (2) (a) RURAL RUNOFF COEFFICIENTS. For rural watersheds, the soil type, land use, and terrain are used to determine runoff coefficients.

The Soil Conservation Service (SCS) has classified about 4,000 major soils found in the United States into four basic hydrologic groups as follows:

- Group A (Low runoff potential). Soils having high infiltration rates even when thoroughly wetted and consisting chiefly of deep, well-drained sands or gravels. These soils have a high rate of water transmission.
- Group B . Soils having moderate infiltration rates when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.
- Group C. Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine texture. These soils have a slow rate of water transmission.
- Group D (High runoff potential). Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly

impervious material. These soils have a very slow rate of water transmission.

The SCS has performed modern soil surveys for many counties in Missouri, classifying the soils by a soil series name. The hydrologic soil group is given in the SCS soil survey for these soils. If a modern soil survey map is not available for the county in question, the hydrologic soil group should be determined by the designer. This may be done by evaluating information available from the soil survey performed by the Materials Division and/or from a site visit and choosing the appropriate hydrologic soil group from the descriptions given above. The district soils and geology technologist may provide assistance in determining the hydrologic soil group.

For smaller projects, the soil series name and hydrologic soil group for soils along the right of way are generally given on the soil survey performed by the Materials Division. For larger projects, this soil survey does not name individual soil series, but rather gives "soil associations" along the right of way. These soil associations are groupings of soil series that have similar physical characteristics; however, the soils in an association may not have the same hydrologic soil group. Additionally, in many instances it will be necessary to determine a hydrologic soil group for soils located some distance from the right of way. In these cases, the hydrologic soil group should be determined by the designer from the SCS county soil survey map, if available, or by evaluating information available from the soil survey performed by the Materials Division and/or from a site visit.

Land use and terrain can be determined by inspection of appropriate topographic maps or by a field inspection. After the hydrologic soil group, land use, and terrain have been determined, the runoff coefficient(s) to be used for estimating the 10-year peak flow for the rural watershed can be found in Table 9-02.2.

TABLE 9-02.2
RURAL RUNOFF COEFFICIENTS FOR 5 AND 10-YR FREQUENCY FLOWS

TOPOGRAPHY AND VEGETATION	SCS HYDROLOGIC SOIL GROUP			
	A	B	C	D
Woodland				
Flat	0.10	0.20	0.30	0.40
Rolling	0.25	0.35	0.40	0.50
Hilly	0.30	0.40	0.50	0.60
Pasture[1]				
Flat	0.10	0.20	0.30	0.40
Rolling	0.16	0.30	0.40	0.55
Hilly	0.22	0.32	0.42	0.60
Cultivated[1]				
Flat	0.30	0.40	0.50	0.60
Rolling	0.40	0.50	0.60	0.70
Hilly	0.52	0.62	0.72	0.82

[1]Flat 0-5% slope; Rolling 5-10%; Hilly 10-30%
Source: Adapted from Ponce (1989)

9-02.5 (2) (b) URBAN RUNOFF COEFFICIENTS. For urban watersheds, the runoff coefficient is primarily a function of land use and watershed slope. Table 9-02.3 gives a range of "C" values for various land use types in urban areas. The value selected by the designer should reflect the watershed slope, with steeper slopes having higher "C" values.

TABLE 9-02.3
URBAN RUNOFF COEFFICIENTS FOR 5 AND 10-YR FREQUENCY FLOWS

DESCRIPTION	RUNOFF COEFFICIENTS
Business:	
Downtown areas	0.70 - 0.95
Neighborhood areas	0.50 - 0.70
Residential:	
Urban single-family	0.30 - 0.50
Urban apartments	0.40 - 0.70
Suburban	0.25 - 0.40
Industrial:	
Light	0.50 - 0.80
Heavy	0.60 - 0.90
Railroad yards	0.20 - 0.40
Parks, cemeteries	0.10 - 0.25
Playgrounds	0.20 - 0.40
Unimproved areas	0.10 - 0.30
Streets and roofs	0.70 - 0.90
Lawns:	
Sandy soil	0.10 - 0.20
Clay soil	0.15 - 0.35
Sodded roadway slopes (1:6 to 1:1)	0.40 - 0.60

Source: Ponce (1979)

9-02.5 (2) (c) DESIGN FREQUENCY. The runoff coefficients given in Tables 9-02.2 and 9-02.3 are appropriate for 5-year or 10-year design frequencies. Estimation of peak flows for less frequent storms requires the use of a higher runoff coefficient because infiltration and other abstractions have proportionately less effect on the amount of rainfall that becomes runoff. To obtain runoff coefficients for other frequency events, the "C" value obtained from Table 9-02.2 is multiplied by a frequency correction factor. The frequency correction factor is 1.10 for the 25-year event, 1.20 for the 50-year event, and 1.25 for the 100-year event. However, the resulting runoff coefficient (original "C" multiplied by the frequency correction factor) may not be greater than 1.0.

9-02.5 (2) (d) COMPOSITE RUNOFF COEFFICIENTS. The land use and soil type may vary from one portion of a watershed to another. In this case an average runoff coefficient, weighted by area, should be used. For example, consider a flat watershed with soils of average infiltration capacity (soil group B) and with 1/3 of the drainage area cultivated (C=0.40) and 2/3 of the drainage area in woodland cover (C=0.20). The weighted average runoff coefficient to be used to estimate the 10-year flood is.

$$C=[(0.40)(1/3)+(0.20)(2/3)]/1.0=0.267 \qquad (2)$$

The frequency correction factor may be applied to the weighted average runoff coefficient to estimate lower frequency flows: in the above example, a "C" value of (1.10)(0.267)=0.294 would be used for the 25-year flood, and a "C" of (1.20)(0.267)=0.320 would be used for the 50-year flood.

9-02.5 (3) TIME OF CONCENTRATION. In order to determine the rainfall intensity used in the Rational Method, the time of concentration of the watershed must be estimated. The time of concentration of a watershed is defined as the time required for water to travel from the most hydraulically distant point of the watershed to the watershed outlet. This is also the time required before the entire watershed begins to contribute flow to the watershed outlet. This characteristic response time of the watershed is used as the duration of the design storm and thus influences the value of rainfall intensity used in the Rational Method.

Note that the location of the most hydraulically distant point in the watershed is a function of travel time and depends on both velocity and distance. The point in the watershed used to determine time of concentration may not necessarily be the point furthest from the watershed outlet.

In general, the time of concentration of urban drainage basins will be shorter than the time of concentration of rural basins. This is true when the natural drainage channels have been altered by storm sewers, main channel straightening, paving or similar modifications.

The time of concentration of an urban watershed may consist of the summation of travel times involved in three different flow regimes; overland flow, ditch or channel flow, and storm sewer system flow. The overland flow regime consists of very shallow sheet flow over the watershed surface. The velocities in overland flow are typically much lower than in the other flow regimes. After a short distance, the sheet flow becomes concentrated in swales or ditches, which begins the open-channel flow phase. The last phase consists of the travel time through the storm sewer system, where velocities are typically greater than through natural ditches or channels. Note that the flow often returns to open-channel flow after exiting the storm sewer system. The time of concentration is then the overland flow travel time plus the storm sewer travel time plus the channel flow travel time.

The sections below provide methods for estimating time of concentration for both rural and urban watersheds. Other methods such as those provided by the Soil Conservation Service (SCS) may be used as deemed necessary or appropriate by engineering judgment.

9-02.5 (3) (a) KIRPICH EQUATION. For small rural watersheds, all flow regimes may be combined into a single equation used to calculate time of concentration. The Kirpich equation is used for these watersheds:

$$t_c = K \cdot L^{0.77} \cdot S^{-0.385} \tag{3}$$

where:

t_c = the time of concentration (min)
K = 0.0078 - ENGLISH [0.0195 - METRIC]
L = the length of the principal watercourse from outlet to divide, ft. [m]
S = the slope between the minimum and maximum elevation, ft./ft. [m/m]

9-02.5 (3) (b) KERBY-HATHAWAY EQUATION FOR OVERLAND FLOW TRAVEL TIME. Overland flow travel time can be calculated using the Kerby-Hathaway equation:

$$t_o = K \cdot (L \cdot n)^{0.47} \cdot S^{-0.235} \tag{4}$$

where:

t_o = overland travel time (min)
K = 0.8262 - ENGLISH [1.439 - METRIC]
L = the overland flow length, ft. [m]
n = roughness coefficient
S = the overland flow slope, ft./ft. [m/m]

The roughness coefficient, n, used in this equation is similar in meaning to that used in Manning's equation for open-channel flow; however, for a given type of surface, the roughness coefficient for overland flow will be considerably larger than for open-channel flow. Typical values for the roughness coefficient are given in Table 9-02.4. The overland flow length should be limited to less than 500 ft. [150 m], and is usually much less. After a short distance, overland flow usually begins to concentrate in swales or ditches, which begins the open channel flow phase.

TABLE 9-02.4
ROUGHNESS COEFFICIENTS FOR OVERLAND FLOW

TYPE OF SURFACE	N
Smooth surfaces (concrete, asphalt, gravel or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover < 20%	0.06
Residue cover > 20%	0.17
Grass:	
Short grass prarie	0.15
Dense grasses (bluegrass, buffalo grass, native grass mixtures	0.24
Bermudagrass	0.41
Range	0.13
Woods	
Light underbrush	0.40
Dense underbrush	0.80

Source: US Army Corps of Engineers (2000)

9-02.5 (3) (c) STORM SEWER TRAVEL TIME. The storm sewer travel time is the summation of travel times in each component of the sewer system between the uppermost inlet and the outlet. These times may be estimated by use of the open channel flow charts presented in Section 9-03. The proper procedure is to solve for the velocity of flow in each component assuming the pipe is flowing at maximum capacity. The velocity of flow is then divided into the length of flow to obtain the travel time. In order to use this method, it is necessary to obtain detailed information on the existing storm sewer system. This information includes conduit sizes, materials, lengths and slopes. If this information is not available or if the structure being designed is not considered important enough to warrant the effort required for such a detailed analysis, the storm sewer travel time may be estimated by use of the Kirpich Equation given in Subsection 9-02.5(3)(a). In this case, the travel time given by the Kirpich Equation should be multiplied by 0.20 to obtain the travel time in the storm sewer system.

9-02.5 (3) (d) CHANNEL FLOW TRAVEL TIME. The travel time in the open channel flow phase can be estimated by first estimating the velocity of flow in the channel. The velocity of flow is then divided into the length of flow to obtain the travel time. For subcritical flow conditions, the velocity should be determined for bank-full conditions at the mid-point of the channel's length. If the channel is improved, the average velocity may be estimated from the open channel flow charts in Section 9-03 or by open channel flow computations. If the channel is natural, the travel time may be estimated by open channel flow computations. Care should be taken when estimating channel flow travel time for supercritical flow conditions. In some cases, the bank-full velocity under supercritical conditions may be too high to provide a reasonable estimate of travel time and a more reasonable estimate of flow depth should be used instead.

9-02.5 (4) RAINFALL INTENSITY. The design rainfall intensity is a function of the storm duration, the design frequency and the geographic location. The storm duration is taken as the time of concentration of the watershed or five minutes, whichever is greater. Knowing the storm duration and the design frequency, the rainfall intensity may be read from the appropriate Intensity-Duration-Frequency (IDF) curve in Figure 9-02.1, 9-02.2, 9-02.3, 9-02.4, 9-02.5, 9-02.6, 9-02.7, 9-02.8, 9-02.9, and 9-02.10. These figures provide IDF curves for each district.

9-02.6 USGS REGRESSION EQUATIONS. The USGS Regression equations were developed by the United States Geological Survey Office in Rolla. Data from stream gage sites were analyzed to determine flood magnitudes with various recurrence intervals. The resulting magnitudes were then related to basin characteristics through a statistical analysis to provide the regression equations. Three sets of equations are currently available: Rural Regression Equations, Urban BDF Regression Equations and Urban Percentage of Impervious Area Regression Equations.

9-02.6 1) RURAL REGRESSION EQUATIONS. These equations were developed in 1995 by the United States Geological Survey in Rolla. Data from 278 gaged sites in Missouri were analyzed to determine flood magnitudes with recurrence intervals of 2, 5, 10, 25, 50, 100 and 500 years. The resulting magnitudes were then related to region, drainage area and average main-channel slope by a statistical analysis to provide the regression equations.

9-02.6 (1) (a) EQUATIONS. For ungaged natural floodflow sites, flood magnitudes having recurrence intervals of 2, 5, 10, 25, 50, 100 and 500 years are computed by using appropriate values of the contributing drainage basin area (A) and slope (S) in the equations shown in Table 9-02.5. The state is divided into three hydrologic regions, each with its own set of equations. The three regions are shown on Figure 9-02.13 and are described as follows:

- Central Lowlands - "Characterized by meandering stream channels in wide and flat valleys resulting in long and narrow drainage patterns with local relief generally between 50 to 150 ft. [15 to 45 m]. Elevations range from about 600 ft. [180 m] above sea level near the Mississippi River to about 1200 ft. [370 m] above sea level in the northwest parts of the region"

TABLE 9-02.5
RURAL REGRESSION EQUATIONS

FREQ. OF FLOOD (Years)	MAGNITUDE OF FLOOD (ft^3/s)	(m^3/s)	STANDARD ERROR OF PREDICTION (%)	AREA LIMITS (mi^2)	(km^2)	SLOPE LIMITS (ft/mi)	(m/km)
Region I							
2	$69.4A^{0.703}S^{0.373}$	$1.87A^{0.703}S^{0.373}$	34.00	0.13 to 11,500	0.34 to 29800	1.35 to 150	0.26 to 28
5	$123A^{0.690}S^{0.383}$	$3.42A^{0.690}S^{0.383}$	32.00	0.13 to 11,500	0.34 to 29800	1.35 to 150	0.26 to 28
10	$170A^{0.680}S^{0.378}$	$4.73A^{0.680}S^{0.378}$	34.00	0.13 to 11,500	0.34 to 29800	1.35 to 150	0.26 to 28
25	$243A^{0.668}S^{0.366}$	$6.70A^{0.668}S^{0.366}$	36.00	0.13 to 11,500	0.34 to 29800	1.35 to 150	0.26 to 28
50	$305A^{0.660}S^{0.356}$	$8.34A^{0.660}S^{0.356}$	38.00	0.13 to 11,500	0.34 to 29800	1.35 to 150	0.26 to 28
100	$376A^{0.652}S^{0.346}$	$10.18A^{0.652}S^{0.346}$	40.00	0.13 to 11,500	0.34 to 29800	1.35 to 150	0.26 to 28
500	$569A^{0.636}S^{0.321}$	$15.01A^{0.636}S^{0.321}$	45.00	0.13 to 11,500	0.34 to 29800	1.35 to 150	0.26 to 28
Region II							
2	$77.9A^{0.733}S^{0.265}$	$1.71A^{0.733}S^{0.265}$	43.00	0.13 to 14,000	0.34 to 36000	1.2 to 279	0.23 to 52.8
5	$99.6A^{0.763}S^{0.355}$	$2.46A^{0.763}S^{0.355}$	36.00	0.13 to 14,000	0.34 to 36000	1.2 to 279	0.23 to 52.8
10	$117A^{0.774}S^{0.395}$	$3.06A^{0.774}S^{0.395}$	34.00	0.13 to 14,000	0.34 to 36000	1.2 to 279	0.23 to 52.8
25	$140A^{0.784}S^{0.432}$	$3.86A^{0.784}S^{0.432}$	32.00	0.13 to 14,000	0.34 to 36000	1.2 to 279	0.23 to 52.8
50	$155A^{0.789}S^{0.453}$	$4.40A^{0.789}S^{0.453}$	31.00	0.13 to 14,000	0.34 to 36000	1.2 to 279	0.23 to 52.8
100	$170A^{0.794}S^{0.471}$	$4.95A^{0.794}S^{0.471}$	32.00	0.13 to 14,000	0.34 to 36000	1.2 to 279	0.23 to 52.8
500	$203A^{0.804}S^{0.503}$	$6.18A^{0.804}S^{0.503}$	34.00	0.13 to 14,000	0.34 to 36000	1.2 to 279	0.23 to 52.8
Region III							
2	$88A^{0.658}$	$1.33A^{0.658}$	34.00	0.48 to 1040	1.24 to 2690	na	na
5	$145A^{0.627}$	$2.26A^{0.627}$	36.00	0.48 to 1040	1.24 to 2690	na	na
10	$187A^{0.612}$	$2.96A^{0.612}$	38.00	0.48 to 1040	1.24 to 2690	na	na
25	$244A^{0.595}$	$3.92A^{0.595}$	41.00	0.48 to 1040	1.24 to 2690	na	na
50	$288A^{0.585}$	$4.67A^{0.585}$	44.00	0.48 to 1040	1.24 to 2690	na	na
100	$334A^{0.576}$	$5.47A^{0.576}$	46.00	0.48 to 1040	1.24 to 2690	na	na
500	$448A^{0.557}$	$7.47A^{0.557}$	54.00	0.48 to 1040	1.24 to 2690	na	na

Source: Alexander and Wilson (1995)

- Ozark Plateaus - "Characterized by streams that have cut narrow valleys 200 to 500 ft. [60 to 150 m] deep, resulting in sharp rugged ridges that separate streams, with local relief generally ranging from 100 to 500 ft. [30 to 150 m]. The drainage patterns are described as dendritic (tree shaped) with main-channel gradients steeper than elsewhere in Missouri, and karst features are locally prominent in much of the region. Elevations (generally) range from 800 to 1700 ft. [240 to 520 m] above sea level"
- Mississippi Alluvial Plain - "A relatively flat area of excellent farmland. Virtually all the area is drained by a series of man-made drainage ditches that slope southward at an average of about 1.5 ft./mile [.28 m/km]. Elevations range from 60 to 90 m (200 to 300 ft) above sea level with local relief seldom exceeding 30 ft. [10 m]".

9-02.6 (1) (b) DRAINAGE AREA. Drainage area (A) in mi^2 [km^2], can be obtained by determining the area contributing surface flows to the site as outlined along the drainage divide on the best available topographic maps.

9-02.6 (1) (c) VALLEY SLOPE. Slope (S) in ft./mile [m/km] is the average slope between points 10 percent and 85 percent of the distance along the main-stream channel from the site to the basin divide. Distance is measured by setting draftsman's dividers at 0.1 mile [0.16 km] spread and stepping along the main channel. The main channel is defined above stream junctions as the one draining the largest area. The elevation difference between the 10- and 85-percent points is divided by the distance between the points to evaluate the slope.

9-02.6 (1) (d) LIMITATIONS OF EQUATIONS. The USGS Rural Regression Equations may be used to estimate magnitude and frequency of floods on most Missouri streams providing the drainage area and slope are within the limits shown in Table 9-02.5.

However, the equations are not applicable for:

- basins where manmade changes have appreciably changed the flow regimen
- the main stems of the Mississippi and Missouri Rivers
- areas near the mouth of streams draining into larger rivers where backwater effect is experienced

9-02.6 (2) URBAN REGRESSION EQUATIONS. These equations were developed in 1986 by the United States Geological Survey in Rolla. Data from 37 gaged sites in both urban and rural locations in Missouri were analyzed to determine flood magnitudes with recurrence intervals of 2, 5, 10, 25, 50, and 100 years. The resulting magnitudes were then related to drainage area and urban basin characteristics to provide the regression equations.

9-02.6 (2) (a) EQUATIONS. Peak discharges can be estimated at urban locations using either of the two sets of equations presented in Tables 9-02.6 and 9-02.7. Both sets give peak discharge as a function of drainage area (*A*) and a characteristic of urbanization: either basin development factor (*BDF*) or percentage of impervious area (*I*). Choice of which set of equations to use should depend on whether it is easier to determine BDF or percentage of impervious area for a given basin. Either set of equations should provide comparable results.

9-02.6 (2) (b) DRAINAGE AREA. Drainage area (*A*) in mi^2 [km^2], can be obtained by determining the area contributing surface flows to the site as outlined along the drainage divide on the best available topographic maps.

9-02.6 (2) (c) BASIN DEVELOPMENT FACTOR. The basin development factor (*BDF*) is determined by dividing the drainage basin into thirds (subareas). Each subarea of the basin is then evaluated for four aspects of urbanization. For each of the four criteria, a value of either 1 (if the subarea meets the criteria) or 0 (if the subarea does not meet the criteria) is assigned. The *BDF* is the sum of the values for each of the four criteria and for each third of the basin. A maximum *BDF* of twelve results when each of the three subareas meets each of the four criteria for urbanization described below:

- Channel Improvements - channel improvements such as straightening, enlarging, deepening, and clearing have been made to at least 50 percent of the main channel and principal tributaries.
- Channel Linings - more than 50 percent of the main channel and principal tributaries has been lined with an impervious material. (Note that the presence of the channel linings also implies the presence of channel improvements.)
- Storm Drains or Storm Sewers - more than 50 percent of the secondary tributaries of a subarea consists of storm drains or storm sewers.
- Curb-and Gutter Streets - more than 50 percent of a subarea is urbanized and more than 50 percent of the streets and highways in the subarea are constructed with curbs and gutters.

The valid range for *BDF* is 0 to 12. Typical drainage-basin shapes and the method of subdivision into thirds are shown in Figure 9-02.11.

TABLE 9-02.6
PEAK DISCHARGE FOR URBAN BASINS BASED ON BASIN DEVELOPMENT FACTOR

FREQUENCY OF FLOOD (Years)	MAGNITUDE OF FLOOD (ft^3/s)	STANDARD ERROR OF ESTIMATE (%)	AREA LIMITS (mi^2)	SLOPE LIMITS (ft/mi)
2	$801A^{0.747}(13\text{-}BDF)^{-0.400}$	32.90	0.25 to 40	8.7 to 120
5	$1150A^{0.746}(13\text{-}BDF)^{-0.318}$	29.40	0.25 to 40	8.7 to 120
10	$1440A^{0.755}(13\text{-}BDF)^{-0.300}$	28.40	0.25 to 40	8.7 to 120
25	$1920A^{0.764}(13\text{-}BDF)^{-0.307}$	27.30	0.25 to 40	8.7 to 120
50	$2350A^{0.773}(13\text{-}BDF)^{-0.319}$	26.50	0.25 to 40	8.7 to 120
100	$2820A^{0.783}(13\text{-}BDF)^{-0.330}$	26.40	0.25 to 40	8.7 to 120

FREQUENCY OF FLOOD (Years)	MAGNITUDE OF FLOOD (m^3/s)	STANDARD ERROR OF ESTIMATE (%)	AREA LIMITS (km^2)	SLOPE LIMITS (m/km)
2	$11.1A^{0.747}(13\text{-}BDF)^{-0.400}$	32.90	0.65 to 100	1.7 to 22
5	$16.0A^{0.746}(13\text{-}BDF)^{-0.318}$	29.40	0.65 to 100	1.7 to 22
10	$19.9A^{0.755}(13\text{-}BDF)^{-0.300}$	28.40	0.65 to 100	1.7 to 22
25	$26.3A^{0.764}(13\text{-}BDF)^{-0.307}$	27.30	0.65 to 100	1.7 to 22
50	$31.9A^{0.773}(13\text{-}BDF)^{-0.319}$	26.50	0.65 to 100	1.7 to 22
100	$37.9A^{0.783}(13\text{-}BDF)^{-0.330}$	26.40	0.65 to 100	1.7 to 22

Source: Becker (1986)

9-02.6 (2) (d) PERCENTAGE OF IMPERVIOUS AREA. The percentage of impervious area (*I*) is the portion of the drainage area that is nonpervious because of buildings, parking lots, streets and roads, and other impervious areas within an urban basin. The variable, *I*, is determined from the best available maps or aerial photos showing impervious surfaces. Field inspection to supplement the maps may be useful.

If the percentage of impervious area cannot be determined directly, a reasonable estimate may be obtained using 7-1/2 minute topographic maps and a relationship between developed area and impervious area. The drainage divide is outlined on the map, then the drainage area is divided into two subareas, open area and developed (urban) area. Open area consists of all undeveloped land, which may include scattered farmhouses and buildings, scattered single-family housing, and paved roads without significant development along the road. Developed areas include single- or multi-family housing structures, large business and office buildings, shopping centers, extensively industrialized areas, and schools. When delineating developed areas, it is important to include those areas devoted to paved parking lots around buildings. Once the amount of developed area has been determined, it can be converted into a percentage developed area (*PDA*) by dividing by the basin drainage area and multiplying by 100. The percentage of impervious area can then be obtained using the following equation:

$$I = 2.03 \cdot PDA^{0.618} \tag{5}$$

The valid range for I is 1.0 percent to 40 percent. The values for both *I* and *PDA* are entered as percents (i.e. *I* = 29 for 29% impervious area and *PDA* = 75 for 75% developed area.)

TABLE 9-02.7
PEAK DISCHARGE FOR URBAN BASINS BASED ON PERCENT IMPERVIOUS AREA

FREQUENCY OF FLOOD (Years)	MAGNITUDE OF FLOOD (ft^3/s)	STANDARD ERROR OF ESTIMATE (%)	AREA LIMITS (mi^2)	SLOPE LIMITS (ft/mi)
2	$224A^{0.793}I^{0.175}$	32.30	0.25 to 40	8.7 to 120
5	$424A^{0.784}I^{0.131}$	29.50	0.25 to 40	8.7 to 120
10	$560A^{0.791}I^{0.124}$	28.60	0.25 to 40	8.7 to 120
25	$729A^{0.800}I^{0.131}$	27.20	0.25 to 40	8.7 to 120
50	$855A^{0.810}I^{0.137}$	26.10	0.25 to 40	8.7 to 120
100	$986A^{0.821}I^{0.144}$	25.90	0.25 to 40	8.7 to 120

FREQUENCY OF FLOOD (Years)	MAGNITUDE OF FLOOD (m^3/s)	STANDARD ERROR OF ESTIMATE (%)	AREA LIMITS (km^2)	SLOPE LIMITS (m/km)
2	$2.98A^{0.793}I^{0.175}$	32.30	0.65 to 100	1.7 to 22
5	$5.69A^{0.784}I^{0.131}$	29.50	0.65 to 100	1.7 to 22
10	$7.47A^{0.791}I^{0.124}$	28.60	0.65 to 100	1.7 to 22
25	$9.64A^{0.800}I^{0.131}$	27.20	0.65 to 100	1.7 to 22
50	$11.20A^{0.810}I^{0.137}$	26.10	0.65 to 100	1.7 to 22
100	$12.78A^{0.821}I^{0.144}$	25.90	0.65 to 100	1.7 to 22

Source: Becker (1986)

9-02.6 (2) (e) LIMITATIONS OF EQUATIONS. The USGS Urban Regression Equations may be used to estimate magnitude and frequency of floods on most urban Missouri streams within the limits shown in Tables 9-02.6 and 9-02.7 provided that the floodflows are relatively unaffected by manmade works such as dams or diversions.

9-02.7 FLOOD HYDROGRAPHS. For certain design problems it may be necessary to determine flood hydrographs associated with the peak discharge for a desired frequency. A hydrograph gives flow rate as a function of time at a particular location in a watershed, usually at the watershed outlet. Two techniques are available for obtaining synthetic hydrographs for ungaged sites. The SCS Unit Hydrograph method is applicable for all watersheds up to 1000 acres [4.0 km^2], and the USGS synthetic hydrograph method is applicable for drainage areas greater than 200 acres [0.8 km^2].

9-02.7 (1) SCS UNIT HYDROGRAPH. Techniques developed by the Natural Resource Conservation Service (NRCS), formerly known as the Soil Conservation Service (SCS), for calculating rates of runoff require the same basic data as the Rational Method: drainage area, a runoff factor, time of concentration, and rainfall information. The SCS method is more sophisticated in that it considers also the time distribution of rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. Additional details on the SCS methodology can be found in the SCS National Engineering Handbook, Section 4 (NEH-4) (SCS, 1985).

9-02.7 (1) (a) RAINFALL-RUNOFF EQUATION AND CONCEPTS. The SCS method is based on a 24-hour storm event. SCS has developed four synthetic rainfall distributions typical of storms for various geographical regions in the United States. The Type II distribution is the appropriate distribution for using the SCS method in Missouri.

The SCS rainfall-runoff equation was developed by the SCS from experimental plots for numerous soil types and vegetative cover conditions. The experimental data consisted mainly of daily rainfall totals on small watersheds, and did not include information on the time distribution of rainfall. The SCS rainfall-runoff equation is therefore used to estimate the depth of runoff resulting from a given depth of 24-hour rainfall assumed to be spatially distributed uniformly over the watershed. The equation is given by:

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \tag{6}$$

where:

Q = accumulated direct runoff, in [mm]
P = accumulated rainfall (potential maximum runoff), in [mm]
I_a = initial abstractions including surface storage, interception and infiltration prior to runoff, in [mm]
S = potential maximum retention, in [mm]

The potential maximum retention, S, is a measure of the maximum amount of water a given watershed could retain, and is a function of land use, interception capacity, infiltration capacity, depression storage, and antecedent moisture. A relationship between I_a and S was also developed from experimental data:

$$I_a = 0.2 \cdot S \tag{7}$$

Substituting the equation for I_a into equation 9, the SCS rainfall-runoff equation becomes:

$$Q = \frac{(P - 0.2 \cdot S)^2}{P + 0.8 \cdot S} \tag{8}$$

It is important to note that while both P and Q are given in units of depth, they actually represent volumes of rainfall and runoff, respectively. This is because the indicated depth of rainfall or runoff is assumed to be applied uniformly to the entire watershed area.

The SCS made additional empirical analyses to estimate the value of S for various watersheds. The principal physical characteristics affecting the relationship between rainfall and runoff are land use, land treatment, soil types and land slope. These characteristics which affect the maximum potential retention are represented by a runoff factor called the curve number (CN), which is related to S by:

$$S = \frac{1000}{CN} - 10 \tag{9}$$

The curve number is an index that represents the combination of hydrologic soil group, land use, hydrologic condition, and antecedent moisture condition. Table 9-08 provides curve numbers for different land uses, treatments and hydrologic conditions; separate values are given for each soil group. The SCS provides methods of adjusting the curve numbers given in Table 9-08 based on varying antecedent moisture conditions; however, for design purposes, average antecedent moisture conditions are normally assumed and the values given in the table can therefore be used for design.

TABLE 9-08a
RUNOFF CURVE NUMBERS – URBAN AREAS[1]

Cover description	Curve numbers for hydrologic soil groups				
Cover type and hydrologic condition	Average percent impervious area[2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]					
Poor condition (grass cover<50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover >75%)		39	61	74	80
Impervious areas:					
Paved Parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm drains (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-or-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only)		63	77	85	88
Artificial desert landscaping (impervious weed barrier desert shrub with 25- to 50mm sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and Business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)		77	86	91	94
Idle lands (CNs are determined using cover types similar to those in Table 9-08c).					

[1] Average runoff condition, and $I_a = 0.2$ S
[2] The average percent impervious area shown was used to develop the composite CNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. If the impervious area is not connected, the SCS method has an adjustment to reduce the effect.
[3] CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.

TABLE 9-08b
RUNOFF CURVE NUMBERS – CULTIVATED AGRICULTURAL AREAS[1]

Cover description			Curve numbers for hydrologic soil groups			
Cover type	Treatment[2]	Hydrologic condition[3]	A	B	C	D
Fallow	Bare soil	-	77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured & terraced (C&T)	Poor	66	74	80	82
		Good	62	71	78	81
	C&T + CR	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C + CR	Poor	62	73	81	84
		Good	60	72	80	83
	C&T	Poor	61	72	79	82
		Good	59	70	78	81
	C&T + CR	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded or broadcast legumes or rotation meadow	SR	Poor	66	77	85	89
		Good	58	72	81	85
	C	Poor	64	75	83	85
		Good	55	69	78	83
	C&T	Poor	63	73	80	83
		Good	51	67	76	80

[1] Average runoff condition, and $I_a = 0.2$ S.
[2] Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.
[3] Hydrologic condition is based on a combination of factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or closed-seeded legumes in rotations, (d) percent of residue cover on the land surface (good>20% and (e) degree of roughness.

Poor: Factors impair infiltration and tend to increase runoff.
Good: Factors encourage average and better than average infiltration and tend to decrease runoff.

TABLE 9-08c
RUNOFF CURVE NUMBERS – OTHER AGRICULTURAL LANDS[1]

Cover description		Curve numbers for hydrologic soil groups			
Cover type	Hydrologic condition[3]	A	B	C	D
Pasture, grassland, or range—continuous forage for grazing[2]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow—continuous grass, protected from grazing and generally mowed for hay		30	58	71	78
Brush—brush-weed-grass mixture with brush the major element[3]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	[4]30	48	65	73
Woods—grass combination (orchard or tree farm)[5]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods[6]	Good	45	66	77	83
	Fair	36	60	73	79
	Good	[4]30	55	70	77
Farmsteads—buildings, lanes, driveways and surrounding lots	--	59	74	82	86

[1] Average runoff condition, and $I_a = 0.2$ S.
[2] Poor: <50% ground cover or heavily grazed with no mulch
Fair: 50 to 75% ground cover and not heavily grazed
Good: >75% ground cover and lightly or only occasionally grazed
[3] Poor: <50% ground cover
Fair: 50 to 75% ground cover
Good: > 75% ground cover
[4] Actual curve number is less than 30; use CN = 30 for runoff computations.
[5] CNs shown were computed for areas with 50% grass (pasture) cover. Other combinations of conditions may be computed fromCNs for woods and pasture.
[6] Poor: Forest litter, small trees and brush are destroyed by heavy grazing or regular burning.
Fair: Woods grazed but not burned, and some forest litter covers the soil.
Good: Woods protected from grazing, litter and brush adequately cover soil.

9-02.7 (1) (b) SCS DIMENSIONLESS UNIT HYDROGRAPH. The SCS unit hydrograph was developed based on analysis of a large number of natural unit hydrographs from a wide range of drainage basin sizes and geographic locations. The SCS unit hydrograph is given in a dimensionless form and provides a standard unit hydrograph shape. Table 9-02.9 gives the ordinates of the SCS dimensionless unit hydrograph.

TABLE 9 -02.9
SCS DIMENSIONLESS UNIT HYDROGRAPH

Time Ratio t/t_p	Discharge Ratio Q/Q_p	Time Ratio t/t_p	Discharge Ratio Q/Q_p	Time Ratio t/t_p	Discharge Ratio Q/Q_p	Time Ratio t/t_p	Discharge Ratio Q/Q_p
0.0	0.000	0.9	0.990	1.8	0.390	3.4	0.029
0.1	0.030	1.0	1.000	1.9	0.330	3.6	0.021
0.2	0.100	1.1	0.990	2.0	0.280	3.8	0.015
0.3	0.190	1.2	0.930	2.2	0.207	4.0	0.011
0.4	0.310	1.3	0.860	2.4	0.147	4.5	0.005
0.5	0.470	1.4	0.780	2.6	0.107	5.0	0.000
0.6	0.660	1.5	0.680	2.8	0.077		
0.7	0.820	1.6	0.560	3.0	0.055		
0.8	0.930	1.7	0.460	3.2	0.040		

Source: McCuen (1996)

Use of the SCS unit hydrograph requires calculation of the unit hydrograph peak discharge and the time to peak. The unit hydrograph peak discharge is given by:

$$Q_p = \frac{K_q \cdot A}{t_p} \tag{10}$$

where:

Q_p = unit hydrograph peak discharge, cfs [m^3/s]
K_q = constant, 484 [2.08]
A = drainage area, mi^2 [km^2]
t_p = time to peak, hrs

The time to peak is assumed to be equal to the basin lag time plus one-half the duration of rainfall. Basin lag time is estimated as 0.6 times the time of concentration, leading to the following equation for time to peak:

$$t_p = \frac{t_r}{2} + 0.6 \cdot t_c \tag{11}$$

where:

t_p = time to peak, hrs
t_r = duration of rainfall (unit hydrograph duration) = 0.133 t_c, hrs
t_c = time of concentration, hrs

9-02.7 (1) (c) APPLICATION OF SCS METHODOLOGY. Unit hydrograph theory depends on the principles of linearity and superposition. Given a unit hydrograph, the runoff hydrograph for a runoff depth other than unity can be obtained by multiplying the unit hydrograph ordinates by the runoff depth using the principle of linearity. The flood hydrograph for a particular storm event can be obtained by dividing the storm event into incremental periods of runoff, then applying the unit hydrograph to each incremental runoff and summing the resulting hydrographs together using the principle of superposition to obtain the total runoff hydrograph.

The unit hydrograph duration (and the corresponding duration of the period of incremental runoff used in applying the unit hydrograph method) is estimated as 0.133 t_c. Since the SCS Type II rainfall distribution has a 24-hour time base, application of the SCS unit hydrograph methodology to typical watersheds by hand requires calculation of runoff hydrographs for a large number of increments. This can be cumbersome and time-consuming and a computer-based implementation is recommended.

9-02.7 (2) USGS SYNTHETIC HYDROGRAPH. A technique for generation of synthetic flood hydrographs for small basins in Missouri was developed in 1990 by the USGS in Rolla. Data from 341 floods recorded at 41 gaging stations located on small rural and urban streams in Missouri were analyzed. An average dimensionless hydrograph applicable to rural and urban basins with drainage areas from 0.25 to 0.40 mi^2 [0.65 to 100 km^2] was developed from this data. This procedure is applicable to flood flows that are not significantly affected by storage or diversions.

The dimensionless hydrograph developed for Missouri is given in Table 9-02.10 and shown graphically in Figure 9-02.12. The dimensionless hydrograph is given in terms of flow divided by peak flow (Q_p) versus time divided by basin lag time (LT). Expansion of the dimensionless hydrograph is accomplished by multiplying each ordinate value (Q/Q_p) by Q_p and each abscissa value (T/LT) by LT, resulting in a flood hydrograph for the desired flood frequency.

TABLE 9-02.10
DIMENSIONLESS HYDROGRAPH FOR SMALL BASINS IN MISSOURI

TIME RATIO (T/LT)	DISCHARGE RATIO (Q/Q_p)	TIME RATIO (T/LT)	DISCHARGE RATIO (Q/Q_p)
0.25	0.11	1.35	0.59
0.30	0.14	1.40	0.53
0.35	0.18	1.45	0.48
0.40	0.23	1.50	0.44
0.45	0.29	1.55	0.40
0.50	0.37	1.60	0.37
0.55	0.46	1.65	0.34
0.60	0.55	1.70	0.31
0.65	0.65	1.75	0.28
0.70	0.74	1.80	0.26
0.75	0.83	1.85	0.24
0.80	0.89	1.90	0.22
0.85	0.95	1.95	0.20
0.90	0.98	2.00	0.19
0.95	1.00	2.05	0.17
1.00	0.98	2.10	0.16
1.05	0.95	2.15	0.15
1.10	0.90	2.20	0.14
1.15	0.84	2.25	0.13
1.20	0.77	2.30	0.12
1.25	0.71	2.35	0.11
1.30	0.65	2.40	0.10

Source: Becker (1990)

9-02.7 (2) (a) PEAK FLOW ESTIMATION. Peak flows of the desired frequency are estimated using the appropriate method given in Subsection 9-02.6.

9-02.7 (2) (b) BASIN LAG TIME. Lag time is usually defined as the time from the center of the rainfall hyetograph to the centroid of the hydrograph. A hyetograph is a plot of rainfall intensity with time. The basin lag time (*LT*) can be estimated using either of two equations:

ENGLISH: $$LT = 1.46 \cdot A^{0.34} \cdot I^{-0.19} \quad (12)$$

METRIC: $$LT = 1.06 \cdot A^{0.34} \cdot I^{-0.19} \quad (12)$$

ENGLISH: $$LT = 0.34 \cdot A^{0.37} \cdot (13 - BDF)^{0.52} \quad (13)$$

METRIC: $$LT = 0.24 \cdot A^{0.37} \cdot (13 - BDF)^{0.52} \quad (13)$$

where:

LT = basin lag time (hrs)
A = basin drainage area, mi^2 [km^2]
I = percentage impervious area as defined in Subsection 9-02.6(2)(d)
BDF = basin development factor as defined in Subsection 9-02.6(2)(c)

The choice of which equation to use should again depend on whether it is easier to determine BDF or percentage of impervious area for a given basin.

9-02.8 DETENTION STORAGE. The traditional purpose of storm drainage systems has been to collect and convey storm runoff as rapidly as possible to a suitable location where it can be discharged. As areas urbanize this type of design may result in major drainage and flooding problems downstream. Under favorable conditions, the temporary storage of some of the storm runoff can decrease downstream flows and often the cost of the downstream conveyance system. Detention storage facilities can range from small facilities contained in parking lots or other on-site facilities to large lakes and reservoirs. This section provides general procedures for detention storage analysis.

An easement must be purchased for any land, outside of the right of way, that will be flooded by water from a detention storage structure.

9-02.8 (1) DATA NEEDS. The following data will be needed to complete storage calculations.

- Inflow hydrograph for all selected design storms. The inflow hydrograph for the detention basin can be determined using the methods in Subsection 9-02.7.
- Stage-storage curve for storage facility.
- Stage-discharge curve for the facility.

Using these data, the inflow hydrograph is routed through the storage facility to develop the outflow hydrograph.

9-02.8 (1) (a) STAGE-STORAGE CURVE. A stage-storage curve defines the relationship between the depth of water and storage volume in a reservoir. The data for this type of curve are usually developed using a topographic map and the conic formula for irregular shaped basins, or the prismoidal formula for trapezoidal basins. The conic formula is expressed as:

$$V_{1,2} = \frac{1}{3} \cdot d \cdot \left(A_1 + A_2 + \sqrt{A_1 \cdot A_2}\right) \quad (14)$$

where:

$V_{1,2}$ = storage volume, ft^3 [m^3], between elevations 1 and 2
A_1 = surface area at elevation 1, ft^2 [m^2]
A_2 = surface area at elevation 2, ft^2 [m^2]
d = change in elevation between points 1 and 2, ft [m]

The prismoidal formula for trapezoidal basins is expressed as:

$$V = L \cdot W \cdot D + (L + W) \cdot Z \cdot D^2 + \frac{4}{3} \cdot Z^2 \cdot D^3 \quad (15)$$

where:

V = volume of trapezoidal basin, ft^3 [m^3]
L = length of basin at base, ft [m]
W = width of basin at base, ft [m]
D = depth of basin, ft [m]
Z = side slope factor, ratio of horizontal to vertical

9-02.8 (1) (b) STAGE-DISCHARGE CURVE. A stage-discharge curve defines the relationship between the depth of water and the discharge or outflow from a storage facility. If the detention facility has both principal and emergency spillways the stage-discharge curve should take both into account. The following equations can be used to help develop the stage-discharge curve.

Sharp-crested weir flow equations for no end contractions, two end contractions, and submerged discharge conditions are presented below, followed by equations for broad-crested weirs, v-notch weirs and orifices, or combinations of these facilities.

9-02.8 (1) (b) 1. SHARP-CRESTED WEIRS. A sharp-crested weir with no end contractions is illustrated in Figure 9-02.14. The discharge equation for this configuration is (Chow, 1959):

ENGLISH: $$Q = [3.27 + 0.4 \cdot (H/H_c)] \cdot L \cdot H^{1.5} \quad (16)$$
METRIC: $$Q = 0.55 \cdot [3.27 + 0.4 \cdot (H/H_c)] \cdot L \cdot H^{1.5} \quad (16)$$

where:

Q = discharge, ft^3/s [m^3/s]
H = head above weir crest excluding velocity head, ft [m]
H_c = height of weir crest above channel bottom, ft [m]
L = horizontal weir length, ft [m]

A sharp-crested weir with two end contractions is illustrated in Figure 9-02.14. The discharge equation for this configuration is (Chow, 1959):

ENGLISH: $$Q = [3.27 + 0.4 \cdot (H/H_c)] \cdot (L - 0.2 \cdot H) \cdot H^{1.5} \quad (17)$$
METRIC: $$Q = 0.55 \cdot [3.27 + 0.4 \cdot (H/H_c)] \cdot (L - 0.2 \cdot H) \cdot H^{1.5} \quad (17)$$

where: Variables are the same as for the previous equation.

A sharp-crested weir will be affected by submergence when the tailwater rises above the weir crest elevation. The result will be that the discharge over the weir will be reduced. The discharge equation for a sharp-crested submerged weir is (Brater and King, 1976):

$$Q_s = Q_f \cdot \left[1 - \left(H_2 / H_1\right)^{1.5}\right]^{0.385} \tag{18}$$

where:

Q_s= submergence flow, ft^3/s [m^3/s]
Q_f= free flow, ft^3/s [m^3/s]
H_1= upstream head above crest, ft [m]
H_2= downstream head above crest, ft [m]

9-02.8 (1) (b) 2. BROAD-CRESTED WEIRS. The equation generally used for the broad-crested weir is (Brater and King, 1976):

<u>ENGLISH:</u> $Q = C \cdot L \cdot H^{1.5}$ (19)

<u>METRIC:</u> $Q = 0.55 \cdot C \cdot L \cdot H^{1.5}$ (19)

where:

Q = discharge, ft^3/s [m^3/s]
C = broad-crested weir coefficient (see Figure 9-02.15)
L = broad-crested weir length, ft [m]
H = head above weir crest, ft [m], measured at least 2.5H upstream of the weir

For weir flow over embankments with sloped sides, a C value of 3.0 should be used. For weir flow over embankments with vertical sides, a minimum C value of 2.6 should be used.

9-02.8 (1) (b) 3. V-NOTCH WEIRS. A V-Notch weir is illustrated in Figure 9-02.14. The discharge through a v-notch weir can be calculated from the following equation (Brater and King, 1976).

<u>ENGLISH:</u> $Q = 2.5 \cdot \tan(q/2) \cdot H^{2.5}$ (20)

<u>METRIC:</u> $Q = 1.38 \cdot \tan(q/2) \cdot H^{2.5}$ (20)

where:

Q = discharge, ft^3/s [m^3/s]
q = angle of v-notch, degrees
H = head on apex of notch, ft [m]

9-02.8 (1) (b) 4. ORIFICES. Pipes smaller than 12 in. [300 mm] may be analyzed as a submerged orifice if H/D is greater than 1.5. For square-edged entrance conditions,

<u>ENGLISH:</u> $Q = 0.6 \cdot A \cdot \left(2 \cdot g \cdot H\right)^{0.5} = 3.78 \cdot D^2 \cdot H^{0.5}$ (21)

<u>METRIC:</u> $Q = 0.6 \cdot A \cdot \left(2 \cdot g \cdot H\right)^{0.5} = 2.09 \cdot D^2 \cdot H^{0.5}$ (21)

where:

Q = discharge, ft^3/s [m^3/s]
A = cross-section area of pipe, ft^2/s [m^2/s]
g = acceleration due to gravity, 32.2 ft/s^2 [9.81 m/s^2]
D = diameter of pipe, ft [m]
H = head on pipe, from the center of pipe to the water surface, ft [m]

9-02.8 (1) (b) 5. CULVERTS. If culverts are used as outlets works, procedures presented in the Culvert Chapter should be used to develop stage-discharge data.

9-02.8 (2) ROUTING CALCULATIONS. The following procedure is used to perform routing through a reservoir or storage facility (Storage Indication or Puhls Method of storage routing).

Routing a flood through a reservoir results in an attenuation of the peak of the inflow hydrograph and an associated change in timing of the peak. Storage of flood waters within the reservoir causes the peak outflow from the reservoir to be lower than the peak inflow, and causes the peak outflow to occur at a later time than the peak inflow. The continuity equation relates the change of storage within the detention storage basin to the inflow and outflow for the basin

$$I - O = \Delta S / \Delta T \qquad (22)$$

where:

I = inflow, ft^3/s [m^3/s]
O = outflow, ft^3/s [m^3/s]
ΔS = change in storage, ft^3 [m^3]
ΔT = change in time, seconds

The Storage Indication method of reservoir routing uses a simple finite-difference form of the continuity equation. For any two points in time, the continuity equation can be written as:

$$\left(\frac{2S_{n+1}}{\Delta T} + O_{n+1}\right) = \left(I_n + I_{n+1}\right) + \left(\frac{2S_n}{\Delta T} - O_n\right) \qquad (23)$$

where:

S = storage

If the values at time step n are known, the only unknowns in equation 20 are on the left-hand side.

Substituting

$$U_{n+1} = \frac{2S_{n+1}}{\Delta T} + O_{n+1} \qquad (24)$$

$$W_n = \frac{2S_n}{\Delta T} - O_n \qquad (25)$$

U is known as the Storage Indication Number. With these substitutions, equation 20 becomes:

$$U_{n+1} = \left(I_n + I_{n+1}\right) + W_n \qquad (26)$$

For the first time step, W_n is calculated using the initial values of S and O, and equation 22. For subsequent time steps the following equation can be used as a shortcut.

$$W_{n+1} = U_{n+1} - 2 \cdot O_{n+1} \qquad (27)$$

The procedure for using the storage-indication method of reservoir routing is as follows:

- Develop an inflow hydrograph, stage-discharge curve, and stage-storage curve for the proposed storage facility.

- Select a routing time period, t, to provide at least five points on the rising limb of the inflow hydrograph.
- Use the stage-storage and stage-outflow data from Step 1 to develop a plot of *U* versus outflow.
- Calculate W_1 using equation 22 and the initial values of *S* and *O*
- Calculate U_{n+1} using equation 23.
- Using U_{n+1} calculated in step 5 pick O_{n+1} from the plot of *U* vs. outflow.
- Using U_{n+1} and O_{n+1} calculate W_{n+1} using equation 24
- Start over at step 5 with n = n+1. Continue repeating until inflow ceases or the outflow peak discharge has been determined.
- From the stage discharge curve, determine the stage for the peak outflow.

9-02.9 REFERENCES

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